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Scholars' Mine

International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

1995 - Third International Conference on Recent Advances in Geotechnical Earthquake Engineering & Soil Dynamics

04 Apr 1995, 2:30 pm - 3:30 pm

Discussions and Replies — Session II

Multiple Authors

DISCUSSIONS AND REPLIES SESSION II

Discussion on paper titled: "Soil-pile-structure during liquefaction on centrifuge," by Sato, Shamoto, and Zhang (Paper No. 2.02)
By: Gregg L. Fiegel, Graduate Research Assistant, Department of Civil and Environmental Engineering, University of California, Davis, California, 95616.

Presented in the paper are results from a dynamic centrifuge experiment conducted to examine the behavior of a 3x3 model pile group founded in a medium dense saturated sand layer. The model was subjected to a moderately sized shaking event with a prototype peak acceleration of 0.3 g. Results showed that the sand layer liquefied over the lower half of the sand layer only. Bending strains in piles were found to be largest near the interface between the liquefied and non-liquefied portions of the sand layer.

Given the results in this paper and another by Sato in *Centrifuge 94* it appears that the Shimizu Corporation has developed a promising centrifuge facility. Several aspects of the centrifuge experiment summarized in the above paper are worthy of discussion.

1.) The authors initially subjected the centrifuge model to a very small shaking event with a peak acceleration of about 0.3 g. The measured acceleration results were then used to evaluate the fundamental vibration characteristics of the soil and pile structure. Other centrifuge researchers should be encouraged to utilize such small strain non-destructive shaking events because results from these preliminary events can be valuable when interpreting results. "Frequency sweep" type input motions can also be used.

2.) In presenting acceleration results the authors used acceleration time histories. Time histories represent important pieces of information; however, it is difficult to determine frequency characteristics from these types of plots. Fourier or acceleration response spectra should be plotted with time histories to aid in the interpretation of the test results.

3.) Silicon oil with a scaled viscosity 30 times that of water was used in the centrifuge experiment to satisfy similitude requirements. Previous research has shown that time dependent phenomenon can be greatly influenced when water is used as the pore fluid in centrifuge liquefaction experiments. Future studies of liquefaction should be conducted with water and higher viscosity fluids to properly evaluate any time dependent effects. This is particularly important when examining mechanisms related to liquefaction induced settlement and lateral spreading.

4.) The authors correctly point-out that the input motion used in a centrifuge experiment should be chosen carefully. It must be remembered that the dynamic behavior of a soil-structure system is dependent on the frequency, intensity, and duration of the input motion. These motion characteristics must each be examined when attempting to understand a mechanism related to soil liquefaction.

Discussion on paper titled "Soil-Pile-Structure during Liquefaction on Centrifuge" by M.Sato, Y.Shamoto, and J.Zhang, Paper No. 2.02)

The objective of the study was to simulate liquefaction phenomena at the soil-pile boundary, and large confining stresses similar to existing in-situ. The scaled test imitated specific conditions with a liquefiable saturated sand stratum underlain by a bearing stratum.

Several important phenomena were demonstrated: liquefaction in a thin layer on a certain depth, redistribution of pore pressure due to permeability of sand, insulating properties of liquefied zone that reduced vibration effect on the structure; concentration of bending strains near the interface between liquefied and non-liquefied zone; stability of group piles against liquefaction.

These important results were achieved using impressive battery of equipment and experimental skills.

The goal of the study was successfully achieved. The only problem is in the philosophy of the study. The problem is in the questions that were not asked. How variations in multiple factors influencing the behavior of the model will influence multiple output parameters? What would happen if: the input signal will be different from one used (different amplitudes, combination of loading cycles, number of cycles)? different values of confining and deviatoric stresses were involved? different pipes were used for modelling of piles? All this and similar questions deal with variability of input factors. The proper answer require development of certain model of behavior of soil-pile-structure system.

At the stage of preliminary design, it would be nice to have a physical model allowing to verify basic design concepts. It is desirable to test any structure against earthquake influence using the loading system similar to described in the paper. But the number of possible experiments may become prohibitive. Application of design of experiment (DOE) methodology may help not only to apply advanced statistical methods to optimize experimental procedures. DOE helps also to formulate the problems.

The discussed paper is an excellent pilot-study of the problem that requires rigorous investigation.

Discussion on paper titled: "Centrifuge modeling of a tilting wall with liquefiable backfill," by Ting and Whitman (Paper No. 2.04)

By: Gregg L. Fiegel, Graduate Research Assistant, Department of Civil and Environmental Engineering, University of California, Davis, California, 95616.

Presented in the paper are results from a series of dynamic centrifuge experiments conducted to examine the behavior of a model earth retaining wall with a liquefiable backfill. The retaining wall used in the experiments was hinged at its base and supported near the top by an tie-back anchor with finite strength. Several experiments were performed using different intensity sinusoidal-like input motions.

The authors showed how the dynamic earth thrust on the wall could be found given the force measured in the tie-back and pore-water pressure measurements near the wall. In addition, the authors estimated the amount of earthquake-induced permanent tilt of the model wall using a lumped mass model with Newmark's sliding block theory. An estimate of earthquake-induced tilt agreed well with that measured.

Also examined in the paper was the effect that the viscosity of the pore fluid had on the results of centrifuge liquefaction experiments. Water was used in a majority of the centrifuge experiments performed; however, in one experiment a glycerol solution with a viscosity 10 times that of water was utilized. The results of this experiment were different from the water experiments. In particular, results showed that pore pressure dissipation was much slower in this model experiment. The authors are correct to point-out that the viscosity of the pore fluid can have a major effect on centrifuge test results. Time dependent phenomenon (i.e. pore pressure dissipation) can be greatly affected by the viscosity of the pore fluid used in centrifuge liquefaction experiments. Researchers must realize that this effect can be very important when examining liquefaction mechanisms (i.e. liquefaction induced settlement, liquefaction induced lateral spreading).

Discussion on paper titled: "Earthquake Input Motions for Physical Model Tests," by G.L. Fiegel, I.M. Idriss, and B.L. Kutter, Paper No. 2.06

By: Dong-Soo Kim, Faculty of Civil Engineering, Korea Advanced Institute of Science & Technology (KAIST), Korea.

The authors presented an interesting centrifugal testing result emphasizing the importance of selecting proper intensity and frequency content of earthquake input motion for physical model tests. The discusser would like to share his experiences on deformational characteristics of soils.

The authors have performed staged model tests where the smallest amplitude earthquakes were applied first followed by successively larger earthquakes. The discusser agreed the general concepts of staged testing, and has used in the resonant column and torsional shear tests. However, deformational characteristics of soils are affected by a previous loading history; stiffness of dry sand increases by cyclic hardening, stiffness of clay decreases by cyclic degradation, and there is a possibility of density variation due to cyclic densification or consolidation during/after earthquake loading. In my opinion, it would be better to show the variation of void ratio using a measured settlement at the surface of each model, and more importantly to verify that model has not been changed after high amplitude earthquake loading. One of the suggested methods is to check the model behavior at a 0.1 g earthquake after high amplitude model testing, particularly after 0.6 g loading where the applied strain level is up to 2.5% (Fig. 14), and then compare it with previous behavior before the high amplitude loading. Repetition of 0.1 g loading will not influence the model adversely because the level of strain amplitude is low. If the comparison is within an allowance, it can be assumed that the model is not permanently altered and further testing can be performed with essentially no effect of past cycling.

The authors presented the effect of frequency content of the input motion on the behavior of clay model. In Figs. 10 and 12, the amplification of the base motion was largest for the DT=0.01 s event and smallest for the DT=0.04 s event at a given level of shaking. For a sand model (Fig. 6), however, the effect of frequency content was not significant. These phenomena can be explained by the variation in soil stiffness with loading frequency. Discusser has investigated the effect of loading frequency on soil stiffness using combined resonant column and torsional shear testing equipment. Shear modulus of cohesive soil increases almost linearly as a function of the logarithm of loading frequency, whereas modulus of dry sand is independent of loading frequency (Kim, 1991). Therefore, clay model for DT=0.01 s (high frequency) event have experienced a much less strain amplitude as shown in Fig. 14, hence a smaller material damping and showed largest amplification.

Figure 14 shows the relationship between maximum shear strain and peak base acceleration. Authors mentioned that the relationship for DT=0.01 s was proportional whereas for DT=0.02 s and 0.04 s, it was nonlinear. It seems a little strange that the relationship for DT=0.01 s is linear because a linear threshold strain for clay is about 0.01% (Kim, 1991, Stokoe and Lodde, 1978), which is an order less than the strain level in the model test. It would be interesting to see the same type of graph for the sand models, and discusser imagines that the effects of frequency content of input motion will be negligible but nonlinearity will start at a less level of shaking.

In the conclusion, authors suggested an effective procedure of conducting dynamic model experiments. One of the steps was to compare the calculated behavior with the observed one in the model test. The discusser agrees this idea but this comparison can not be found in the text. From the discusser's view, it would be interesting to investigate the effects of strain amplitude, confining pressure, and loading frequency on shear modulus and damping ratio of model soils using dynamic tests, then to predict the model behavior using analytical procedures, and finally to compare the predicted behavior with the observed behavior.

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Discussion on paper titled: "Earthquake Input Motions for Physical Model Tests", By Fiegel, Idriss and Kutter, Paper No. 2.06.

By: NaiHsin Ting, Engineer, China Engineering Consultants, Inc., Taipei, Taiwan. R.O.C.

The authors developed a very insightful testing program, with various time steps (DT) and acceleration levels, to investigate the dynamic characteristics of the soil models. The results demonstrated the importance of understanding the dynamic characteristics of a centrifuge soil model.

The paper suggests that it is necessary to investigate the dynamic characteristics of the soil models. The discussor, however, thinks that exploration of the dynamic characteristics of the shaker-model system may be of comparable importance when simulating actual earthquakes in centrifuges.

The acceleration records presented in this paper indicate that the predominant period of the shaker-model system was about 0.3 second, both with sand and clay models. Figure 2 shows that original Santa Cruz motion has two predominant periods at about 0.15s and 0.3s. However, the high frequency components were attenuated in the shaker-model system. The predominant period of the measured motion was at about 0.3s to 0.4s for the DT = 0.02s events. Figure 4, showing the measured Santa Cruz motions with various predominant periods, further suggests that the predominant period of the system was about 0.3s.

The authors highlight that the dynamic characteristics of the shaker-model system may have substantial influence on the actual earthquake motions applied to models. The discussor agrees with the authors that using dummy models various trial tests to investigate the discrepancy of the dynamic characteristics between the calculated and observed motions. The discussor would also like to ask for the authors' opinion, about the possibility of changing the frequency content of the input signal, knowing the discrepancy between the calculated and observed natural frequency, to get closer simulation of the desired motion applied to the model.

Discussion on paper titled: "Critical Acceleration Levels for Free Standing Bridge Abutments", By Fishman, Richards & Divito, Paper No. 2.12.

By: NaiHsin Ting, Engineer, China Engineering Consultants, Inc., Taipei, Taiwan, R.O.C.

The authors presented interesting laboratory observations to highlight the seismic reduction in bearing capacity of the foundation soil beneath bridge abutments.

This paper highlights that the threshold acceleration for movement of gravity wall bridge abutment is related to base sliding as well as seismic reduction of bearing capacity. The authors describe that seismic bearing capacity reduction is strongly dependent on the level of acceleration, the shear transfer between the wall footing and the soil, and the shear strength of the foundation soil. They also indicate that the seismic reduction of bearing capacity induces rotation of such gravity retaining wall structures during shaking, and the wall failure by rotation is quite common in earthquake damage reports and laboratory tests.

Inertia effect, which can be considered together with the acceleration level, may have been included by the authors as a factor affecting the seismic reduction of the bearing capacity. However, the discussor would like to further point out that the inertia thrust on the wall causes a rotation moment about the wall heel, which induces an incremental vertical load on the model wall footing during horizontal excitation. This extra vertical load increases substantially with the acceleration level, and should also be included as a factor contributing to the wall failure by rotation, in addition to the seismic reduction of bearing capacity.

The discussor's very rough estimation, assuming the height/width ratio of the retaining wall is about 2 to 3, shows that the vertical load resisted by the soil beneath the toe of the free-standing wall may increase substantially during horizontal shaking. The increased load results from the inertia of the soil and of the wall itself. The vertical load on the wall footing due to the earth thrust may double during shaking if k_h is at about 0.1 to 0.15; the load due to the wall itself may double if k_h is at about 0.3 to 0.8.

Based on Figure 6, the observed threshold acceleration level of Model II was about 0.2g in Table 3. Therefore, the theoretically predicted threshold acceleration levels are slightly larger than the observed ones for both Models II and III. Furthermore, the amount of overprediction increases with the acceleration level. Such comparisons confirm the previous discussion qualitatively.

In conclusion, the theoretical predictions may be more close to the model test observations if the inertia moments (from both the dynamic earth and wall thrusts) are included.

Discussion on paper titled: "Behavior of saturated sand models under principal stress axes rotation in shake table tests", By E. Yanagisawa & Jafarzadeh, Paper No. 2.13.

By: NaiHsin Ting, Engineer, China Engineering Consultants, Inc., Taipei, Taiwan, R.O.C.

The authors presented some interesting results of an extensive series of one-dimensional dynamic model tests on saturated loose and medium sand models. The discussor, however, would like to share his thoughts regarding the model test results.

In general, the discussor assumes that the test results presented in this paper are in model scales, instead of in prototype scales. The discussor numbers the twelve tests in Table 1 as L1, L2, ···, L6 and D1 to D6. In which the letters "L" and "D" indicate loose and medium dense models, respectively. The digits indicate the individual tests from top to bottom in Table 1.

Various correlations were proposed based upon a total 72 sets of obsevrations from the 12 tests. This paper presents 4 out of the total 72 sets of results in Figures 3 and 4, namely P2 in Test L2 (Fig. 3a), P2 in Test L4 or L3 (Fig. 3b), P3 in Test D2 (Fig. 4a) and P1 in Test D6 (Fig. 4b).

The discussor finds that in Figures 3 and 4:

1. The "zero" horizontal acceleration in Fig. 3a is at about 145 gals. This might due to a tilt of the accelerometer prior to shaking or other unknown reasons.
2. Three out of the four (excess) pore pressure ratios u_r in these figures were significantly less than 100% at their peak values, while liquefaction (Fig. 3b) or cyclic mobility (Figs. 3a and 4a) are indicated by accelerograms. Such results may be caused by various factors, such as inaccurate sensor locations, settlement of the transducers during test, improper saturation of pore pressure transducers, etc. However, it is still difficult for the discussor to figure out what happened to the u_r ratios in Figs. 3b and 4a, as the end-of-shaking u_r values were very low (less than 2/3) while transducers were located deep in the models.
3. In Figure 3b, u_r started at about 48% prior to input motion and reached to about 65% after about 4 load cycles. The discussor assumes that an incorrect vertical scale was used in this plot.

The results presented in this paper show that the soil models were properly prepared and the tests were well conducted. As long as pore pressure transducers are properly installed and well saturated, in cyclic tests such as L1, L2, D1 and D2, it is rational to observe negative u_r and double cycling of u_r during initial load cycles (Ting, 1993). Double cycling of u_r indicates that the soil experienced a dilation-contraction-dilation-contraction cycle during each shearing cycle (Whitman and Ting, 1993). Such variation is also observed in Fig. 4a. The above characteristics sometimes do not appear in similar tests with less proper model preparations.

The correlations proposed in this paper are interesting. However, based upon the above discussions, a revisit to the test data with proper interpretations will be helpful towards the development of more proper correlations based on the test results.

Reference

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Discussion on paper titled "Behavior of saturated sand models under principal stress axes rotation in shake table tests" by E.Yanagisawa and F.Jafarzadeh, paper #2.13)

By: Sergey Drabkin, Research Engineer of Civil Engineering Department, Polytechnic University, Brooklyn, New York, USA

The objective of the study was to establish experimental correlations between excess pore water pressure ratio (u_r) developed in sand box and shearing strain of sand (γ) caused by vibration of that box produced by shake table.

The paper demonstrated well designed equipment, thorough specimen's preparation technique, and poorly defined plan of experiment.

Experiment should start with defining input factors and output parameters. Here independent variable input factors are specimen's density (loose and medium dense); shape of vibrational signal (sinusoidal or random); vibrational amplitude varied on two levels for sinusoidal signal and on four levels for random signal; depths of layers (six levels). Such input factors as mean effective stress (p'), deviatoric stress, shear work, and normalized shear work were not independent but calculated using independent factors. Such input factors as number of cycles and frequency in sinusoidal excitation were constant.

The directly measured output parameters were pore water pressure, acceleration, and horizontal displacement. The calculated output parameters were u_r and γ.

This accounting of parameters allows to demonstrate the weaknesses in planning of experiment presented in the paper. Table 1 gives applied input waves. It is not clear why sinusoidal waves with maximum accelerations 80 and 300 Gal were used for testing of loose specimens, but for testings of medium dense specimens the chosen amplitudes were already 69 and 239 Gal, respectively. That lack of logic is characteristic for the paper.

The earthquake types of excitations were considered as qualitative factors (as different patterns of loading in Fig. 3 and 4). But they were characterized also with such quantitative characteristics as acceleration of random loading in Fig. 8 without defining what accelerations authors had in mind. In Fig. 7, twenty seven maximum accelerations are shown for 4 medium dense models. Either each layer was considered separately that gives twenty four points only, or what?

The choice of levels of sinusoidal vibration amplitudes looks quite random, and cannot be compared with earthquake amplitudes. It is impossible to compare the output parameters because no two input factors are alike (Fig. 7 and 8).

Only obvious effects were demonstrated: pore pressure existed while vibration was applied (Fig. 3 and 4).

Correlations were demonstrated between such two output parameters as u_r and γ_{max} in Fig. 5 and 6, maximum shear stress ratio and γ_{max} in Fig. 9, and u_r and shear work in Fig. 10. They have limited value because they were given for mixed values of input factors.

Organization of experimental study of such a complex and multi-variable problem requires knowledge of principles of multifactorial experimental design (see for example, Box et al., 1978, and Box and Draper, 1987).

Introduced by Eq.1 dynamic shear stress obviously depends on confining pressure that is mentioned but not shown. The applicability of this parameter in a current form for the following presentation of strain energy is questionable.

The theoretical discussion of the problem should have started from defining the boundary conditions and applied excitation. Then the problem could be solved by methods of dynamic theory of elasticity.

The conclusions of the paper are also not clear. What does it mean: "dynamic shear strain is an applicable parameter for correlating the generated u_r in loose or dense models"? The logarithmic correlation in Fig. 5 & 6 depends on many other input factors. It may be linear as well for such scattering of data.

No plot supports also the statement that u_r is independent from confining pressure. I think that conclusion is wrong. Permeability of water and rapid equalizing of pore pressure between thin layers could have hidden the influence of confining pressure. I agree with conclusion that u_r is independent from stress path.

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Discussion on paper titled: " Dynamic Deformational Characteristics of Rockfill Materials from Laboratory Test, In-Situ Test, and Earthquake Motion Analysis," by T. Iwashita, N.Yasuda, A.Nakamura, and O. Takeda, Paper No. 2.22

By: Dong-Soo Kim, Faculty of Civil Engineering, Korea Advanced Institute of Science & Technology (KAIST), Korea.

The authors presented an interesting comparison of dynamic deformational characteristics of rockfill materials determined by large-scale cyclic triaxial tests, in situ geophysical tests, and response analyses of earthquake motions. The discusser, however, has several questions regarding the authors' arguments.

The authors have presented the shear wave velocity profile predicted from the laboratory tests in Fig. 2. The discusser tried to check the relationship of shear wave velocity versus depth using the equations 1 to 5 and assumed that G_o is in same units as P_{at}, but when K=0.5, the calculated in situ shear wave velocity was much less than the one shown in Fig.2.

The authors have presented that shear moduli of rockfill material, G_o, at small strain are proportional to $(\sigma'_m)^{0.94}$ and $(\sigma'_m)^{0.73}$ for the principal stress ratios of 1.0 and 0.5, respectively. Based on test results on sands (Hardin, 1978, Lewis, 1990), however, the slope of G_o versus σ'_m is much flatter, and is almost proportional to $(\sigma'_m)^{0.5}$. It would be nice to see the summary plots of Log(G_o) versus Log (σ'_m) for rockfill materials, and the predicted shear wave velocity profile shown in Fig. 2 is significantly affected by the slope of this plot.

The authors presented the variations in normalized shear modulus, G/G_o, and damping ratio with strain amplitude determined by cyclic triaxial tests. From the discusser's experiences, material damping ratio of sand is significantly affected by number of loading cycle (It decreases dramatically in the first ten cycles), and therefore, it would be better to clearly specify the number of cycle in Fig. 4 at which damping ratio is determined.. At strain amplitude below 10^{-5}, no damping ratio is plotted in Fig. 4. The discusser expects this due to the difficulty in measuring accurate stress-strain hysteresis loops at small strains. With the modification of motion monitoring system, the discusser have found that hystertic damping ratio of sands still exists and is independent of strain amplitude at strains as low as $6 * 10^{-7}$ (Kim and Stokoe, 1994).

The authors have obtained total damping ratios (h_t) of the dams from the frequency response functions of the observed earthquake motions by half power methods, and estimated the radiation dampings by subtracting the hysteretic dampings from the total damping ratios. In the discusser's opinion, accurate estimation of h_t for the first to third resonance frequencies by half power method would be difficult and the half power method sometimes overestimates the damping ratio at large strains where frequency response is not symmetric. It would be interesting to show the typical frequency response curve which used in the damping calculation. In addition, it is not clear whether each mode of acceleration is used separately or only maximum acceleration in the time history is used, in the estimation of strain amplitudes for the corresponding total damping ratios of different modes.

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Reply of discussion on paper (No.2.02) titled: "Soil-Pile-Ptructure during Liquefaction on Centrifuge" Discussion by Gregg L. Fieggel

Reply by the writers: Masayoshi Sato, Yasuhiro Shamoto & Jian-Min Zhang, Institute of Technology, Shimizu Corporation

The writers are grateful to the discusser for pointing out the several important aspects of the dynamic centrifuge model test technology. The writers is also pleased to have an opportunity to show our following opinions and give some complementary statements.

1) The natural frequency of an actual soil-pile-structure system may decrease significantly due to stiffness reduction of the soil when it is shaken by a strong earthquake. Considering strong non-linearity of the soil, it is necessary to determine the intrinsic frequency of a soil-pile-structure corresponding to the initial soil stiffness before it encounters a destructive shaking. It is not only benefit to correctly evaluating the results of dynamic centrifuge model tests, but also to reasonably analyzing earthquake response of the system by numerical methods of simulation. A further study on this aspect has been performed and will be presented for publication.

2) In the presenting results of acceleration response of the model soil-pile-structure system to an input shaking, the writers not only show acceleration time histories in Fig.5, also provide the frequency response functions measured from the preparatory non-destructive shaking test and shown in Fig.4. It is worthy of note that response of the soil and pile structure to a shaking undergoes drastic changes from pre-liquefaction to post-liquefaction, therefore, the two stages of liquefaction (including process of excess pore water pressure build-up) and subsequent post-liquefaction should be distinguished in the evaluation of frequency characteristics. For the liquefaction stage the evaluation of frequency characteristics seems to have not definite meaning because soil stiffness is undergoing drastic changes during this process, and in addition, it is much difficult to obtain Fourier or acceleration response spectra with a satisfactory degree of accuracy because the data that can be measured are very limited for each short time interval analyzed. During the continuing shaking after the appearance of complete liquefaction, the soil stiffness is nearly zero and accordingly the responses of the soil and pile structure become very small and their predominant frequency approaches nearly zero, which has been confirmed by the experimental facts of a series of 1-g and centrifuge shaking tests of the soil-pile-structure system performed by the writers.

3) The writers agree with the discusser that time dependent effects concerning permeability of the model soil or the pore fluid in centrifuge liquefaction experiments are particularly important when revealing mechanism related to liquefaction induced settlement and lateral spreading. In the presenting study silicon oil with a viscosity 30 times as that water was used to fill in the voids in the sand for simulating the permeability, which corresponded to a prototype sand layer with high permeability or the coefficient of permeability of 3×10^{-2} *cm/sec*. It will be difficult to reasonably reproduce the actual liquefaction behavior if water is used in the centrifuge model tests becuase the permeability of the soil is too largely evaluated.

4) The writers also agree with the discusser that the dynamic behavior of a soil-pile-structure system depends on the frequency, intensity, and duration of the input motion. The results of the experimental studies for the effect of these motion characteristics on the behavior of the soil-pile-structure system conducted by the writers will be presented for publication.

Reply of discussion on paper (No.2.02) titled: "Soil-Pile-Structure during Liquefaction on Centrifuge"

Reply by the writers: Masayoshi Sato, Yasuhiro Shamoto & Jian-Min Zhang, Institute of Technology, Shimizu Corporation

The writers wish to thank the discusser for his interest in their paper and valuable discussion. The writers agree with the discusser that there indeed exist various factors influencing the behavior of the model soil-pile-structure system, and application of DOE methodology is much benefit to optimizing experimental procedures and formulating the problems. However, it is worthy of note that before the DOE methodology is used, we need to know what are the main factors controlling the behavior of the model. Namely, application of DOE methodology should be based on the understanding of the behavior of the model in a certain degree when applicable experimental conditions and procedures are designed.

Paper No. 2.04
Reply by N-H. Ting
China Engineering Consultants, Inc., Taipei, Taiwan

The writers appreciate the discussion by Fiegel. The discussor pointed out the importance of using viscous pore fluid in centrifuge modeling of soil liquefaction.

In dynamic centrifuge modeling there is a conflict between the time scalings of dynamic perturbation and of the diffusion process, where time in the model is reduced by N and N^2, respectively. For tests modeling soil liquefaction such conflict brings about a much faster dissipation of the excess pore pressure generated during cyclic shearing. The dissipation may take place during the process of pore pressure build-up in the model. Therefore, the pore pressure build-up will be less substantial in the model.

Two alternatives are usually adopted to reduce the effect of the conflict, either to increase the viscosity of the pore fluid or to reduce the particle size of the soil so as to reduce its permeability (Steedman and Zeng, 1995).

The two series of tests with a pore fluid ten times more viscous than water showed substantial different characteristics of pore pressure build-up than those observed in similar tests using water as the pore fluid. Results show that the pore pressure dissipation during earthquake may influence the occurrence of soil liquefaction in some critical cases.

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Reply to discussion by Dr. Dong-Soo Kim on paper : "Dynamic Deformation Characteristics of Rockfill Materials from Laboratory Test, In-Situ Test, and Earthquake Motion Analysis", by T. Iwashita, N. Yasuda, A. Nakamura, and O. Takeda, (Paper No. 2.22.)

We wish to thank Dr. Kim for his comments on our paper.

We carried out the re-tests of the large-scale cyclic triaxial test for rockfill materials of Miho Dam after contribution of our paper. Axial micro displacement was measured by more accurate linear variable differential transformer for displacement (Capacity is 5mm and accuracy is less than $\pm 0.5\%$) installed on the specimen cap. Cyclic load was applied at 0.1 cycles per second and repeated 12 times for each step. And the data of the 10th cycle were used for the analysis. The re-test result show the relationship of the shear modulus G_0 at infinitesimal strain and mean effective principal stress σ'_m in Fig.1. The shear modulus of rockfill materials, G_0, is proportional to $(\sigma'_m)^{0.558}$ and $(\sigma'_m)^{0.582}$ for the principal stress ratio (K) of 1.0 and 0.5, respectively.

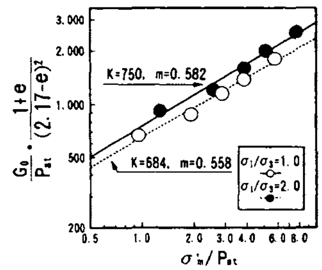


Fig.1 Relationship of G_0 and σ'_m for rockfill materials of Miho Dam

The left of Fig.2 shows the distribution of S-wave velocity, Vs, with depth below the dam surface, D, obtained from laboratory re-test for Miho Dam. The results of in-situ geophysical explorations is also shown in this figure. We show also the case for Oya Dam in the right of Fig.2. The distribution of Vs from the laboratory tests agreed with that from the in-situ geophysical exploration for Miho Dam and Oya Dam.

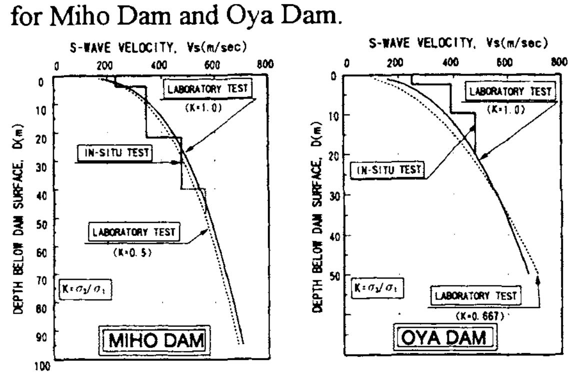


Fig.2 Comparison of Vs from laboratory test and in-situ test for Miho Dam and Oya Dam

We have obtained total damping ratio h_t of rockfill dams from the frequency response functions of the observed earthquake motions by half power method, and estimated radiation damping ratio. The estimation of h_t by half power method is somewhat rough calculation, but has cleared the frequency dependent characteristics of the radiation damping ratio. Figure 3 shows the frequency response function of the largest shaking case (earthquake No.③) of the six observed earthquake motions. The frequency response is not so asymmetrical despite at large strain.

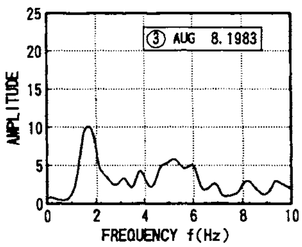
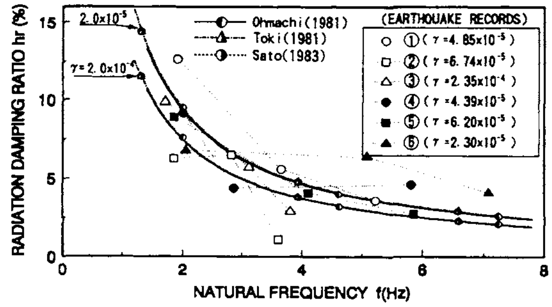


Fig.3 Frequency response function of observed earthquake No.③ for Miho Dam

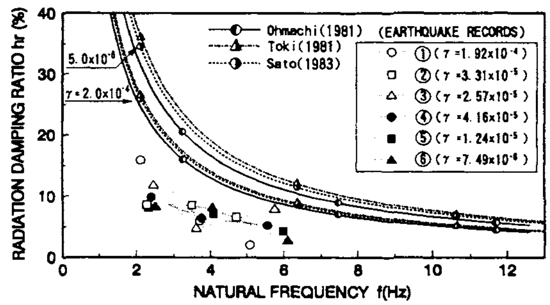
In our paper, the radiation damping ratio of each mode was estimated by subtracting the same value of hysteresis damping ratio for the same shear strain γ at every mode from the total damping ratio of each mode. The shear strain, however, greatly differs at each mode. We calculated the shear strain γ_i for the i-th mode from the rate of Fourier spectrum amplitude of the shear strain for the i-th natural frequency against the peak amplitude, as expressed by :

$$\gamma_i = \gamma \cdot \frac{F_{\gamma,i}}{F_{\gamma\,max}} \qquad \left(\gamma = \frac{a_{max}}{(2\pi f_1)^2 \cdot H} \right)$$

where, $F_{\gamma,i}$ is Fourier spectrum amplitude of the shear strain for the i-th natural frequency, $F_{\gamma\,max}$ is the peak amplitude of Fourier spectrum of shear strain and a_{max} is the maximum acceleration at dam crest. We, thus, estimated the hysteresis damping ratio $h_h(\gamma_i)$ for the shear strain γ_i of the i-th mode from the result of laboratory tests. The frequency dependency of the radiation damping ratio obtained by subtracting $h_h(\gamma_i)$ of each mode from the total damping ratio $h_{t,i}$ is shown in Figs.4 for Miho Dam and Oya Dam.



Miho Dam



Oya Dam

Fig.4 Radiation damping ratio with natural frequency for Miho Dam and Oya Dam